Appendix C
Swing Gate Design Example

The opening for the closure in this example is 29.50 ft wide by 12.30 ft high.

Load Cases:

In accordance with EM 1110-2-2502, consideration shall be given to load cases I1 through I4. An additional case for the gate in any position, subjected to dead load only, is added for checking hinge design.

Case I1, Design Flood Loading. Gate is closed; water on the unprotected side is at the design flood elevation; water is at or below sill surface on protected side. Design stresses shall not be greater than 5/6 of stresses allowed in AISC (1989).

Case I2, Maximum Flood Loading. Same as case I1 except that water level is to top of gate on unprotected side. Design stresses shall not be greater than 1.11 times the stresses allowed in AISC (1989).

Case I3, Earthquake Loading. Water is at usual level (non-flood condition) on unprotected side; earthquake-induced forces are acting. (Note: This case is applicable to support structures only).

Case I4, Short-Duration Loading. Gate is either open, closed, or in between and is subjected to construction and/or wind loads. Design stresses shall not be greater than 1.11 times the stresses allowed in AISC (1989).

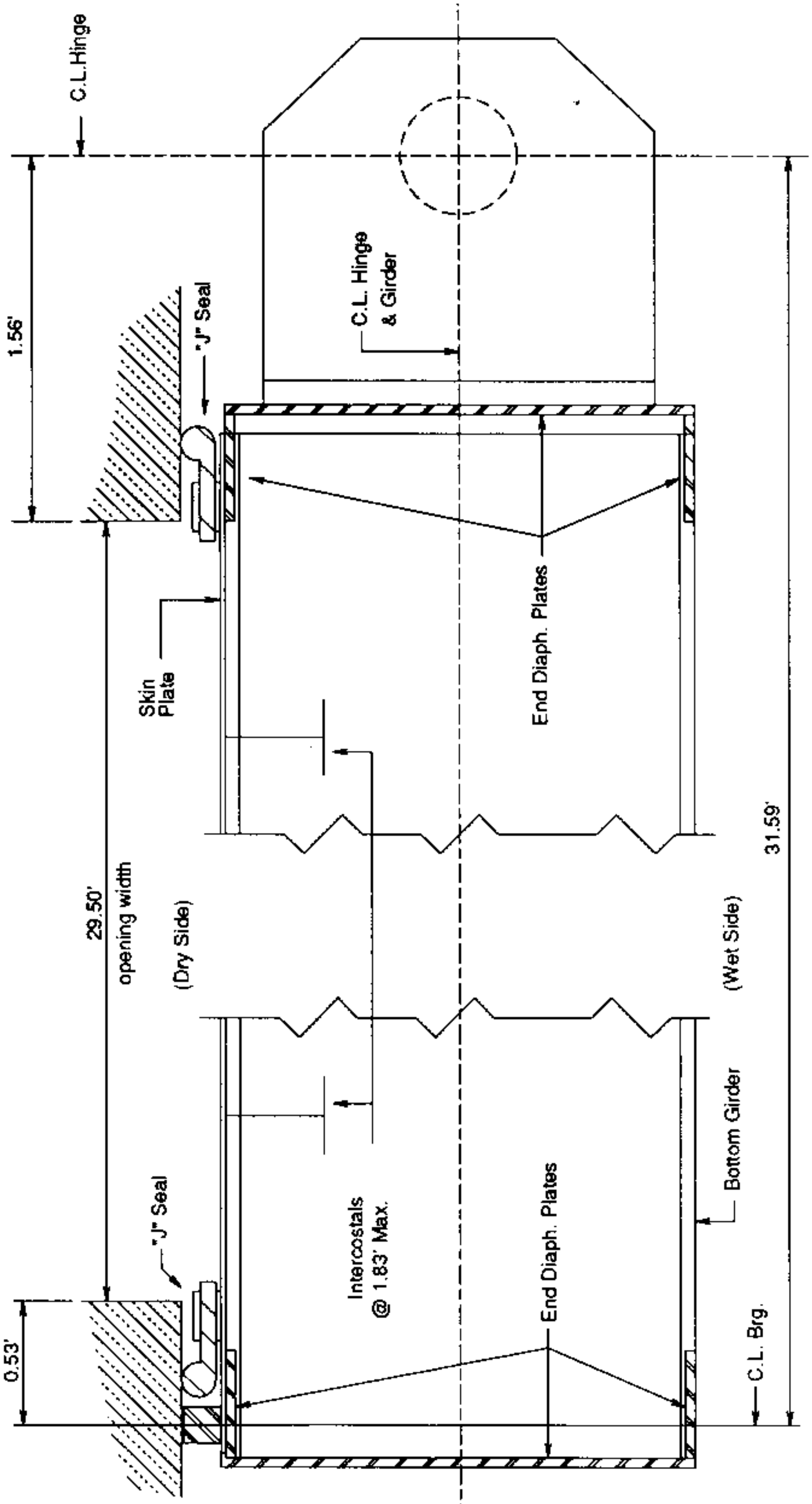
Case I5, Gate Swinging. Gate in any position, subjected to dead load only. Design stresses shall not be greater than 5/6 of stresses allowed in AISC (1989).

In this example, cases I1 and I3 are not significant and skin plate, intercostals, and girders are designed for case I2 with design stresses not greater than 1.11 times stresses allowed in AISC (1989). Case I4 is applicable for design of latching devices. Case I5 is applicable to hinge design.

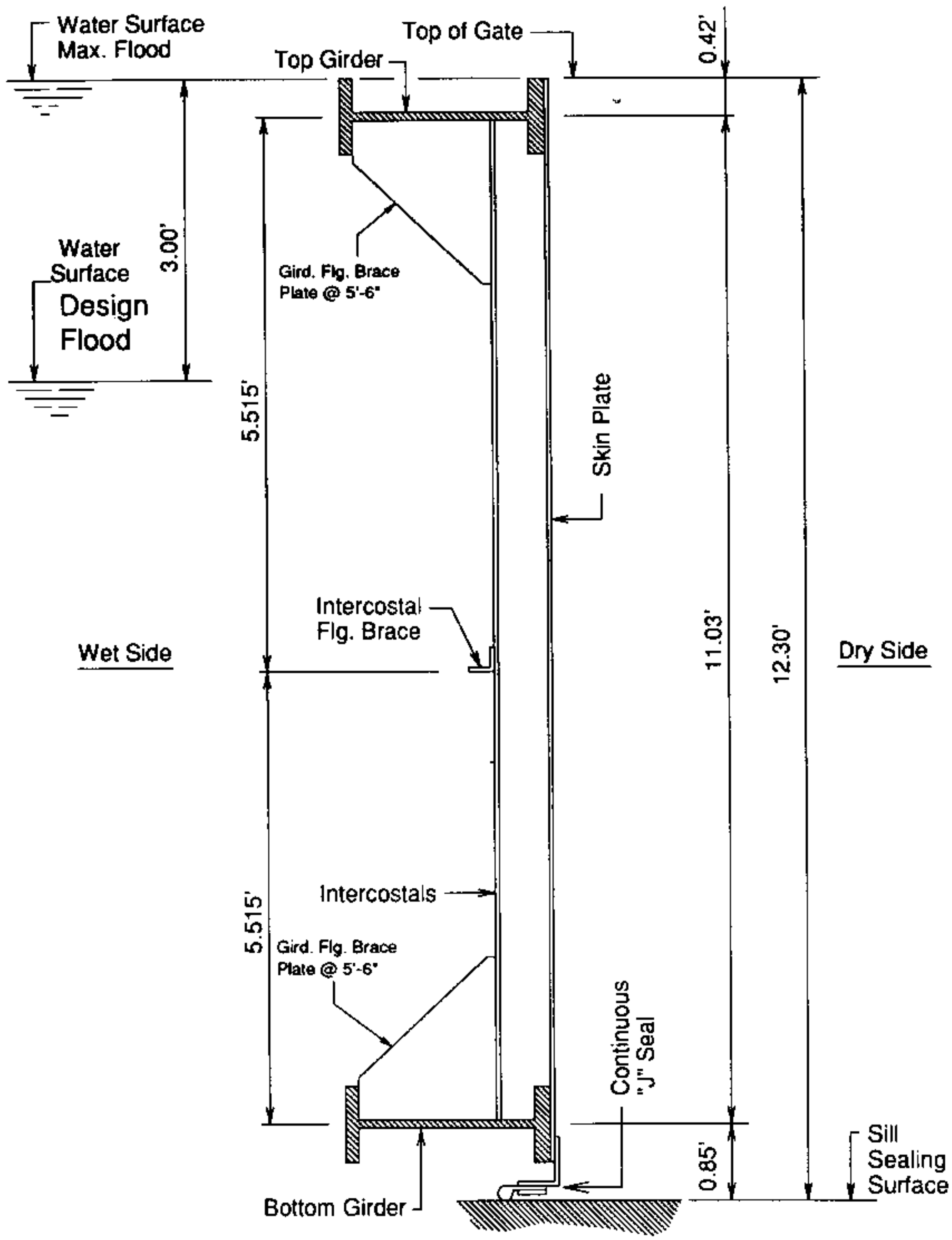
The skin plate is designed as a fixed end beam, spanning between intercostals. In order to ensure that the flat plate theory is applicable, deflection will be limited to 0.4 of thickness.

The intercostals are designed as simple beams spanning between girders.

Girders are designed as simple beams, spanning between hinges on one side and wall bearings on the other side of the opening.



HORIZONTAL SECTION THRU GATE



VERTICAL SECTION THRU GATE

SKIN PLATE DESIGN; LOAD CASE I2 (Material: A36 Steel)

Assuming a 9" flange for the bottom girder, the hydrostatic pressure 6" above the flange is:

$p=0.0625(12.3-0.85-0.375-0.5)=0.6609ksf=0.00459ksi$

$M=pb^2/12=0.00459(22)^2/12=0.1851k\text{-}in$

$F_b=1.11x0.75F_y=1.11x0.75x36=30ksi$

$t_{min\text{-}stress}=(pb^2/2F_b)^{1/2}$

$t_{min\text{-}stress}=[0.00459(22)^2/(2x30)]^{1/2}=0.1924"$

$defl.=pb^4/384EI;\ E=29000,\ I=t^3/12,\ defl.=0.4t$

$0.4t=12pb^4/384Et^3,\ t^4=pb^4/12.8E$

$t_{min\text{-}defl.}=[0.00459(22)^4/(12.8x29000)]^{1/4}=0.2320"$

USE 1/4" SKIN PLATE

For Tall Gates More Than One Thickness May Be Required.

INTERCOSTAL DESIGN: LOAD CASE I2

Material: A36 Steel. Equations & Tables In Parentheses Are From The Specification In AISC (1989).

Load, shear, and moment diagrams for intercostals are shown on page C-5. Trial Section is shown on page C-6.

$M=5.839x22/12=10.705k\text{-}ft=128.46k\text{-}in$

$b_f/2t_f=4.00/(2x0.27)=7.41$

$65/(F_y)^{1/2}=65/6=10.83>7.41$ Compact Section (Table B5.1)

$L_b=5.515'=66.18",\ L_b/r_T=66.18/1.03=64.25$

$L_c=76b/(F_y)^{1/2}=76x4/6=50.67<66.18$ (F1-2)

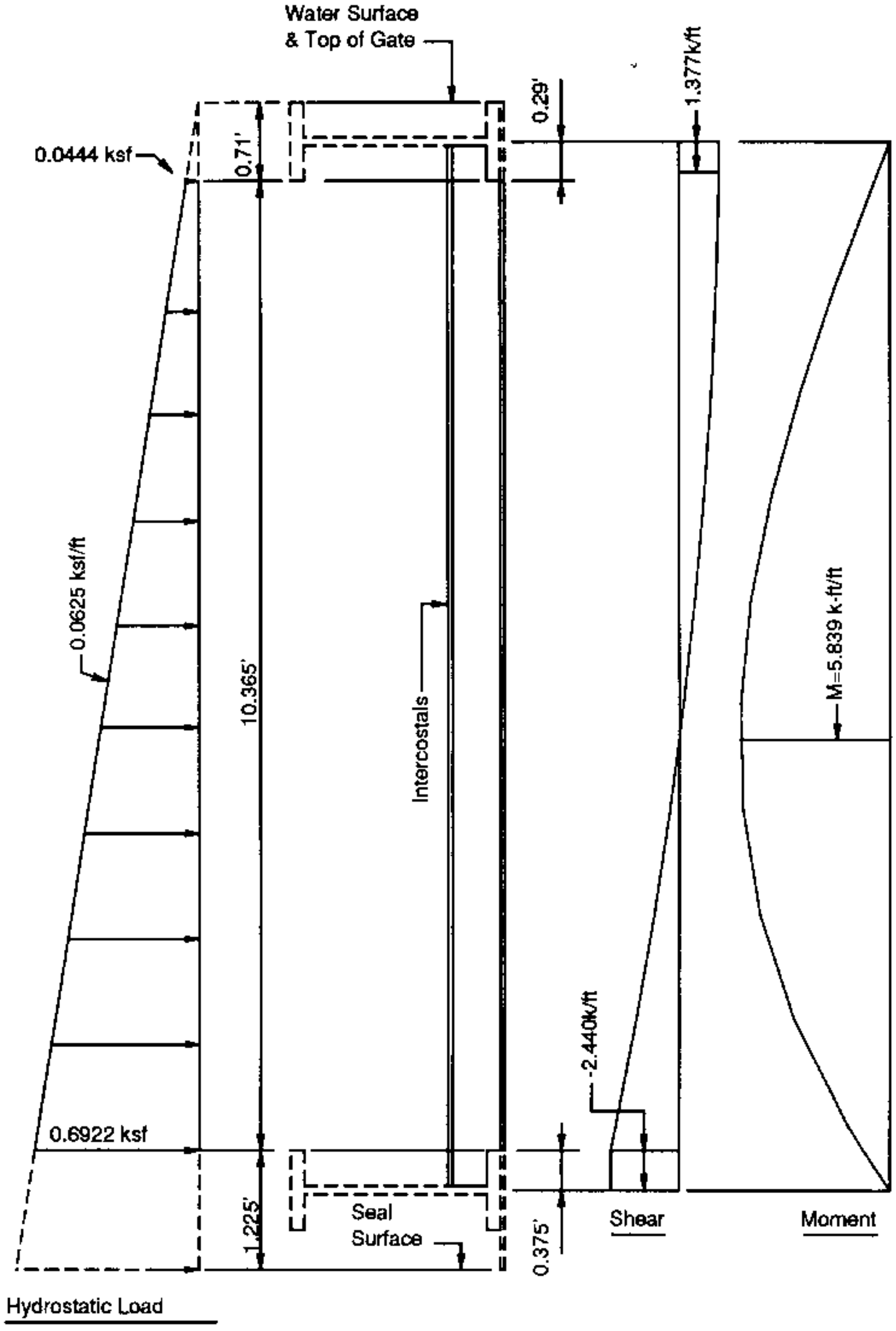
$F_b=[2/3-F_y(L_b/r_T)^2/(1530x10^3)]F_y$ (F1-6)

$F_b=[2/3-36(64.25)^2/(1530x10^3)]36=20.50ksi$

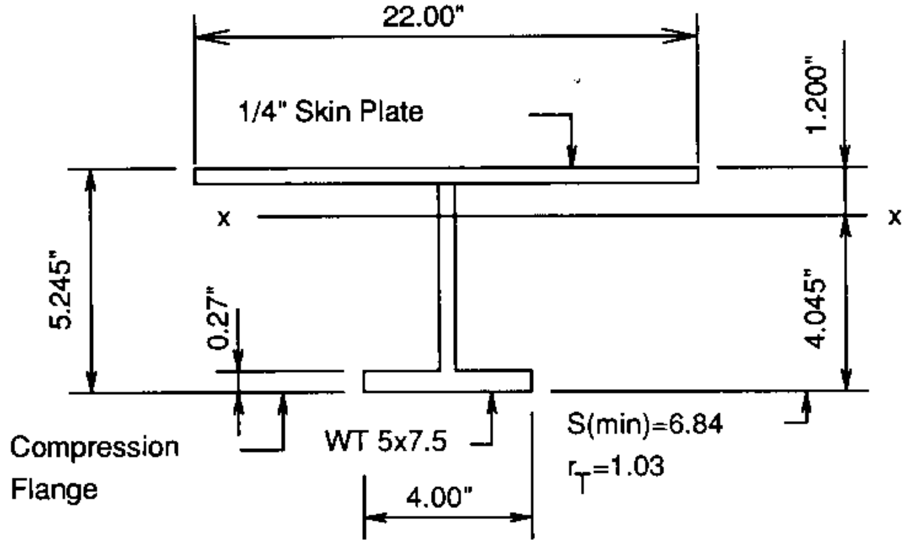
$1.11F_b=22.78ksi$

$f_b=M/S=128.46/6.84=18.78ksi<22.78ksi$ o.k.

USE WT 5x7.5 FOR INTERCOSTALS. MAX. SPACING 1'-10"



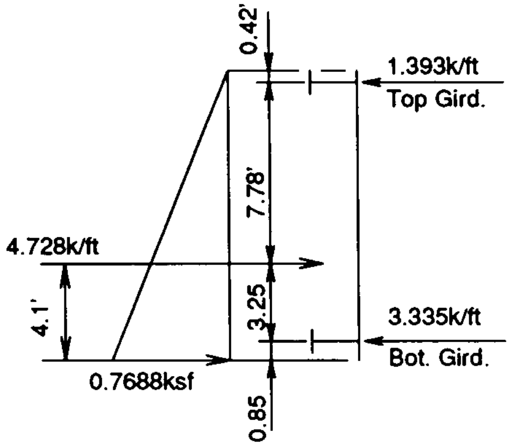
LOAD, SHEAR, & MOMENT DIAGRAMS FOR INTERCOSTALS



INTERCOSTAL SECTION

Girder Design

(Mat'l. A36 Steel, Equation and Table Numbers In Parentheses Are From Specification In AISC (1989).)



GIRDER LOADS

Bottom Girder

$M=3.335(31.59)^2(1/8)$

$M=416.04\text{k-ft}=4992\text{k-in}$

Trial Section W 24x84

$I_x=2370$, $S_x=196$

$d/A_f=3.47$, $b_f=9"$, $F_y=36\text{ksi}$

$b_f/2t_f=5.9$, $d/t_w=51.3$

(Table B5.1)

$65/(F_y)^{1/2}=10.83>5.9$

$640/(F_y)^{1/2}=106.67>51.3$

Flange & Web are compact

$L_b=66"$, $L_c=76b_f/(F_y)^{1/2}=114">66"$ (F1-2)

$F_b=1.11\times0.66\times F_y=26.67\text{ksi}$, $f_b=M/S_x=4992/196=25.47\text{ksi}<F_b$

USE W 24x84

See page C-18 for design with the effect of axial load & bending due to the diagonal

Top Girder

$M=1.393(31.59)^2(1/8)=173.76\text{k-ft}=2085\text{k-in}$

Trial Section W 24x55

$I_x=1350$, $S_x=114$, $d/A_f=6.66$

$b_f/2t_f=6.9$, $d/t_w=59.7$, $L_b=66"$

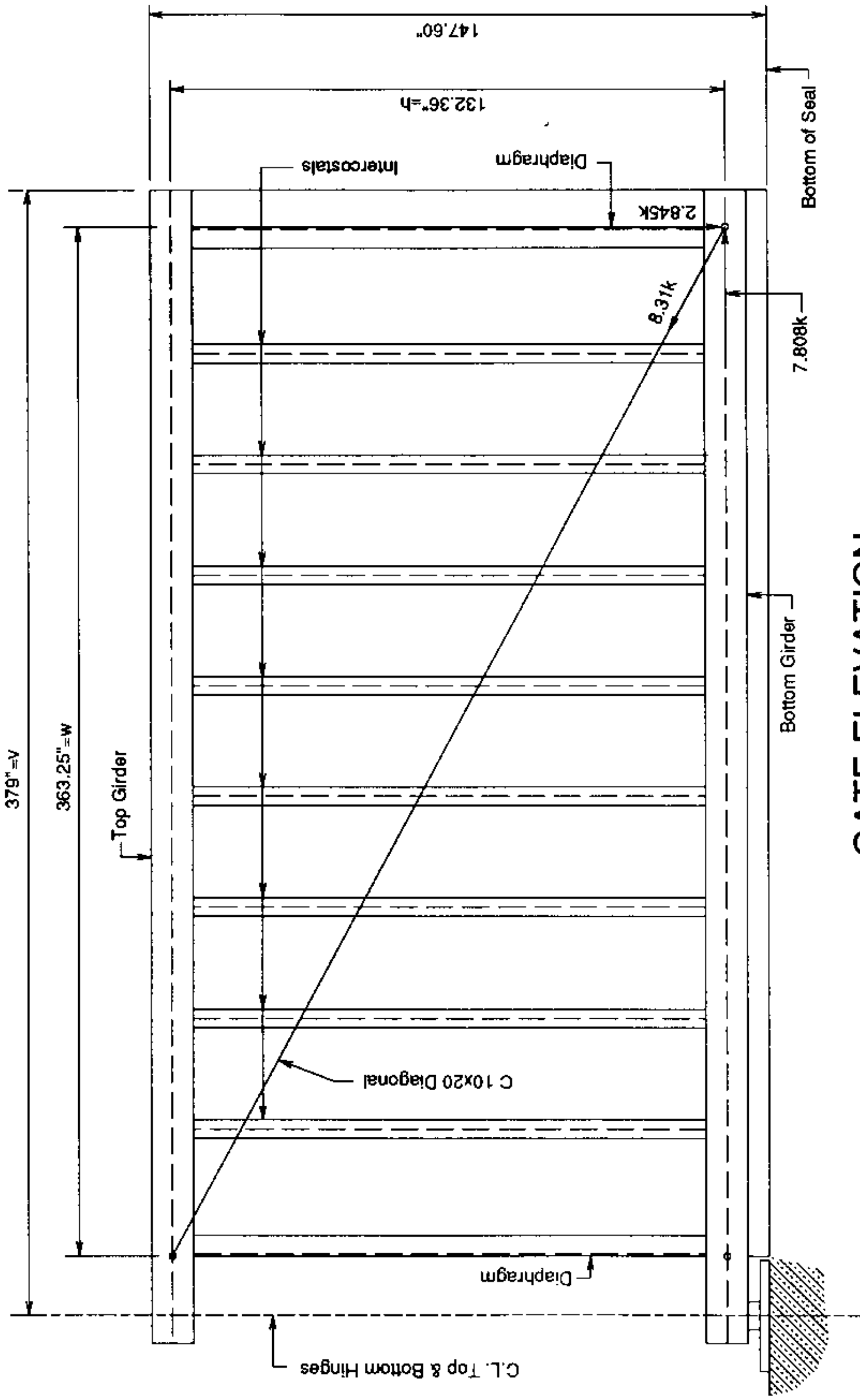
Flange & Web are compact, see Table B5.1.

$L_c=20000/(6.66\times36)=83.42">66"$ (F1-2)

$F_b=1.11(2/3)(36)=26.67\text{ksi}$

$f_b=2085/114=18.29\text{ksi}<26.67\text{ksi}$

USE: W24x55



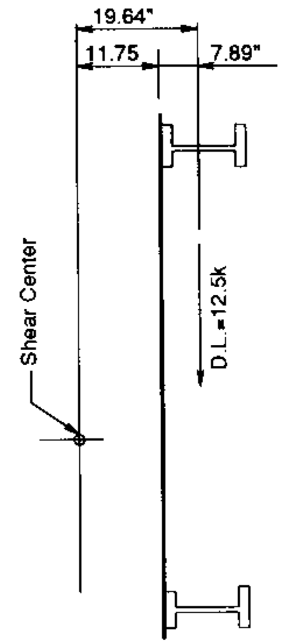
GATE ELEVATION

SHOWING DIAGONAL LOAD DUE TO D.L. TORSION

(See Page B-9 For Torsional Deflection & Diagonal Stress Calculations)

STRESS & DEFLECTION DUE TO DEAD LOAD TORSION

Methods For Determining The Shear Center And The Torsional Stiffness Of The Gate Are Presented In EM 1110-2-2703



End View

T=Torsion=12.5x19.64=245.5k-in

Z=Distance From Hinges To C.G. Of D.L.

Z=16.198'=194.38"

TZ=245.5x194.38=47720

A=Area C 10x20 Diagonal=5.88

A'=1/8 Of Sum Of Girder & Diaph. Areas

A'=10.11

$L=[(132.36)^2+(363.25)^2]^{1/2}=386.61"$

Qo=Torsional Stiffness Of Gate =2442

t=Dist. From C.L. Of Skin To C.G. Of Diagonal

t=22.97"

Ro=2wt/vL=2x363.25x22.97/(379x386.61)

Ro=0.1139

R=A'Ro/(A+A')=10.11x0.1139/(5.88+10.11)

R=0.072

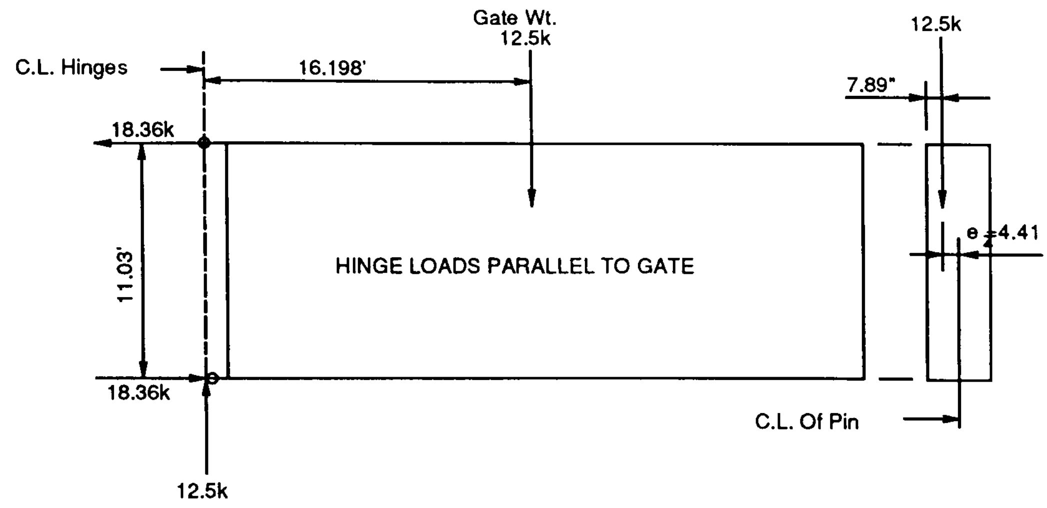
Q=RRoEAhv/L=Torsional Stiff. Diagonal

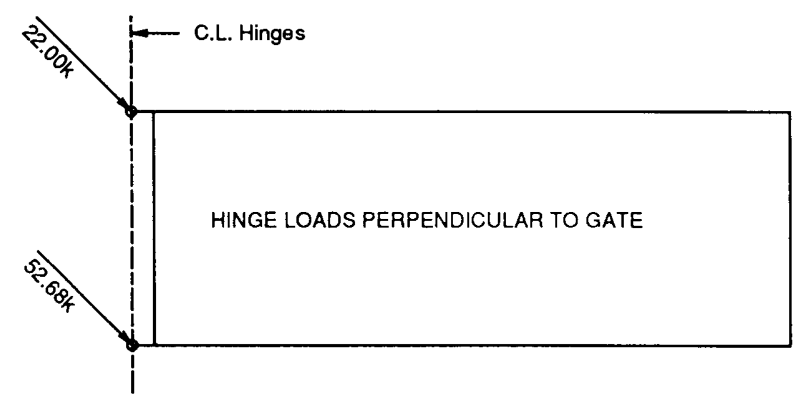
Q=181312, Q+Qo=183754

$\triangle$ =TZ/(Q+Qo)=47720/183754=0.26"=Torsional Defl. Of Gate.

Tensile Stress In Diagonal=RE$\triangle$/L=1.41ksi

Tensile Load In Diagonal=1.41x5.88=8.31k





BOTTOM HINGE PIN

P_y =52.68k, P_x =18.36k, P_z =12.5k, e_z =4.41"

Assuming a 1.75" base plate, Z = 7.85"

$My = P_y Z + P_z e_z = 468.66$ k-in

$M_x = P_x Z = 144.13$ k-in

$M_r = \sqrt{M_x^2 + M_y^2} = 490.32$ k-in

d= Pin Diameter = 5", Mat'l. A276, Fy=55 ksi

A =19.63, I = 30.68, S = 12.27, r =1.25, kL/r=2x7.85/1.25=13

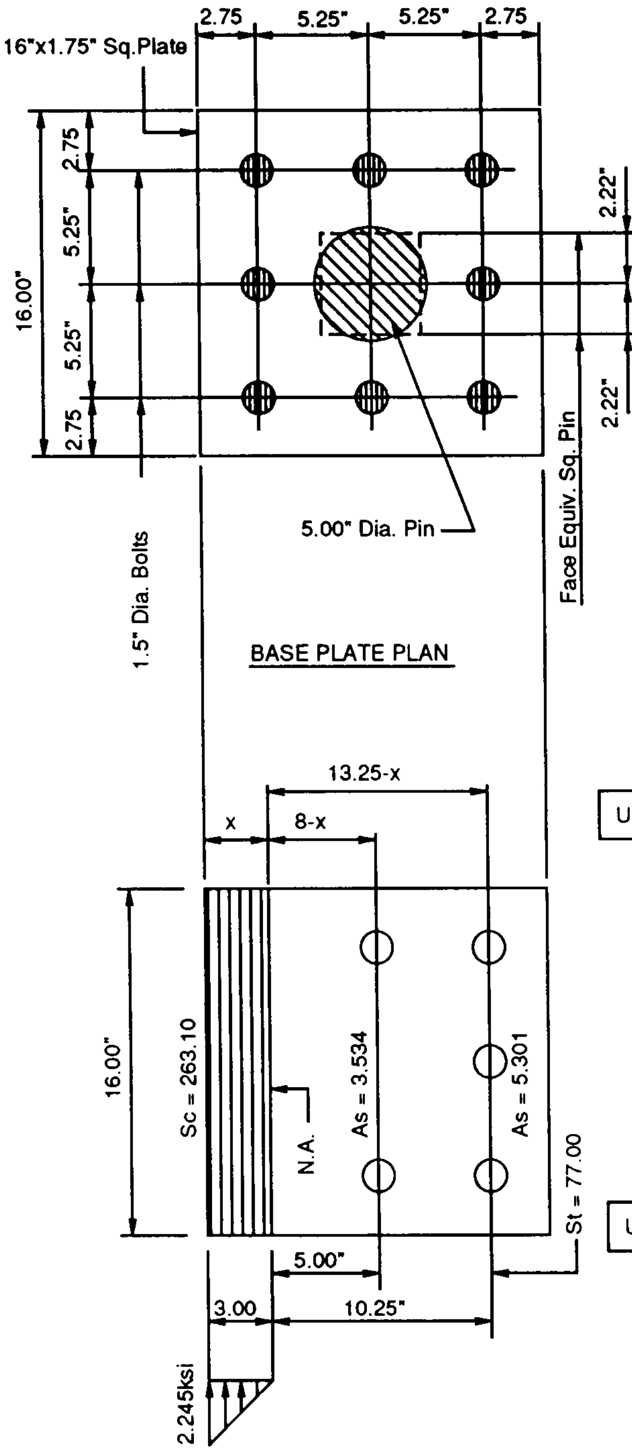
1.11Fb=1.11x0.75Fy=45.83ksi, 1.11Fa = 35.43ksi (Using AISC (1989))

f_a =12.5/19.63=0.637ksi, f_b = 490.32/12.27 = 39.96 ksi

$f_a/1.11F_a + f_b/1.11F_b = 0.890<1.000$

USE 5" DIAMETER PIN

BASE PLATE & BASE PLATE BOLTS:



Brg. Press.

$P = \sqrt{Px^2 + Py^2} = 55.79k$

Moment at Bottom of Plate;

M=55.79x9.6+12.5x4.41=590.71k-in

(neglect 12.5k vertical load)

Calculate Section Properties

At Bottom Of Plate.

$8x^2 = 3.534(8-x) + 5.301(13.25-x)$

$8x^2 + 8.835x - 98.51025 = 0$

x = 3.00"

Bolt Mat'l. F593, Alloy 316, Fy=30ksi

$I_{N.A.} = 16(3)^3/3 + 3.534(5)^2 + 5.301(10.25)^2 = 789.29in^4$

Sc = 789.29/3 = 263.10

St = 789.29/10.25 = 77.00

f_c =590.71/263.10 =2.245ksi

f_t = 590.71/77.00 =7.672ksi

F_t=1.11x0.6x30=20ksi

F_v=1.11x0.4x30=13.33ksi

f_v=55.79/(8x1.767)=3.95ksi

USE 8-1.5" DIA. BOLTS F593

Moment in Plate at Edge of

Equiv. Square Pin =

0.5x2.245x3.00x4.78=16.10k-in

$S_{PL} = 1(1.75)^2/6 = 0.510$

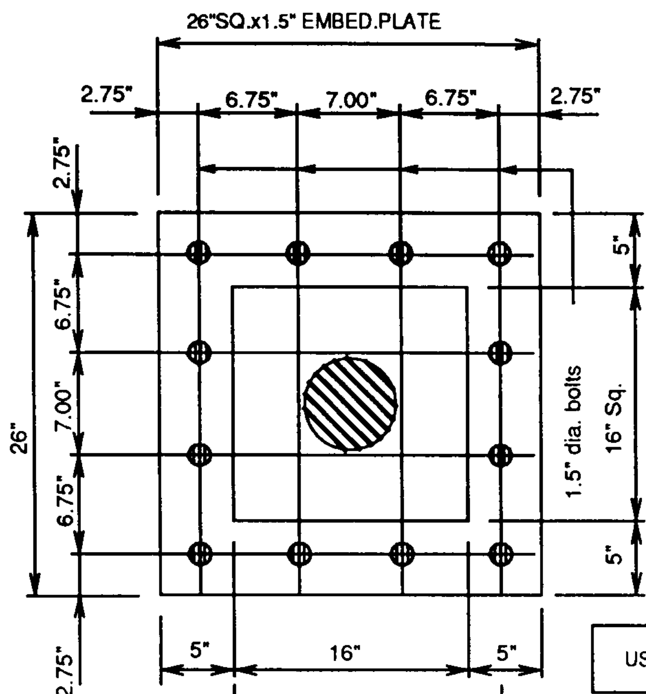
Mat'l. ASTM A276, Fy=55ksi

1.11Fb = 1.11x0.75x55 =45.83ksi

f_b= 16.10/0.510 = 31.57ksi<45.83ksi

USE 16" SQ. x1.75" A276 PLATE

EMBEDDED PLATE & BOLTS: (Bolt Mat'l. F593, Alloy 316. Plate A36)



PLAN

Properties of Transformed Section At Base Of Embed. Plate:

$n = Es/Ec = 10$

$13x^2 = 35.34(16.5\text{-}x) + 70.68(23.25\text{-}x)$

$13x^2 + 106.02x - 2226.42 = 0$

$x = 9.63"$

$I_{N.A.} = 22519.21\text{in}^4$

$Sc = 2338.44$, $St = 1653.39$

$M = 55.79x11.1 + 12.5x4.41 = 674.39\text{k-in}$

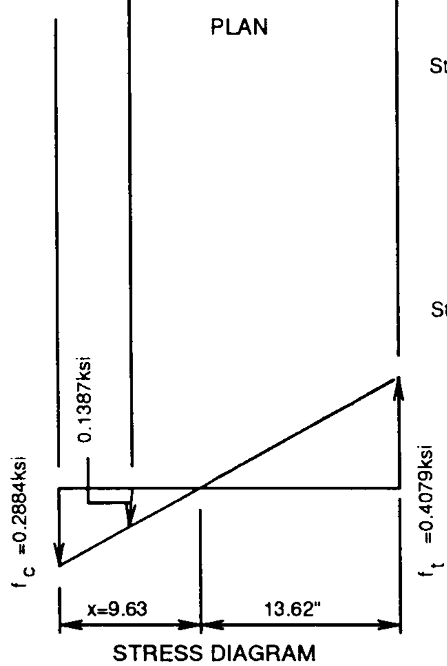
$f_c = M/Sc = 0.2884\text{ksi}$

$f_t = M/St = 0.4079\text{ksi}$

Bolt Tension $= nf_t = 4.079\text{ksi}$

Ft = 20ksi, Fv = 13.33ksi

USE 12-1.5" DIA. F593 BOLTS



STRESS DIAGRAM

Stress At Edge Of Base Plate:

$S_{PL} = (1.5)^2/6 = 0.375$

$M = 0.5(0.2884+0.1387)(5)(2.79) = 2.98\text{k-in}$

$f_b = 2.98/0.375 = 7.95\text{ksi}$

Stress At Edge Of Equiv. Pin:

$M = 0.5(0.2884)(9.63)(8.57) = 11.90\text{k-in}$

Combined Section Modulus for Two Plates:

$S = 0.375 + 0.510 = 0.885$

$f_b = M/S = 11.90/0.885 = 13.45\text{ksi}$

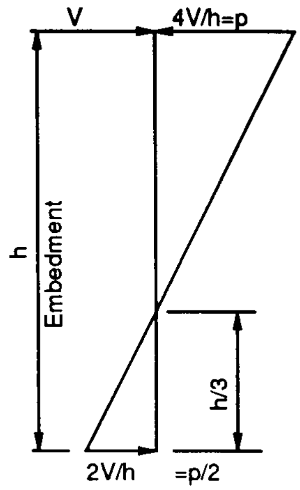
USE 26" SQ. x 1.5" A36 PLATE

See Page C-13 For Bending Stress In Embedded Bolts & Bolt Bearing Stress On Concrete.

BOLTS IN CONCRETE

Using Maximum Allowable Concrete Bearing Stress, Find The Minimum Required Bolt Embedment:

f'_c=3ksi (28 day concrete compressive strength)



Assumed Pressure Distribution

h=Min. Embed.=4V/p

$F_p=0.35f'_c=1.05$ksi From Sect. J9 Of AISC (1989)

d = Bolt Dia. =1.5" F_p(for Case I2)=1.11x1.05=1.167ksi

p = dF_p = 1.5x1.167 = 1.75k/in

V = 55.79/12 =4.65k/Bolt

h = 4x4.65/1.75 = 10.63" = Min. Embed. Req'd.

Actual Embedment = 20">10.63" o.k.

Calculate Flexural Stress In Bolt:

M = Max. Moment = 4Vh/27 = 4x4.65x10.63/27 = 7.323k-in

S = Bolt Sect. Mod. = 0.3313

1.11(0.75F_y) = 1.11x0.75x30 = 25ksi = F_b

f_b= M/S = 7.323/0.3313 = 22.10ksi<25ksi o.k.

CALCULATE BOND STRESS DUE TO BOLT TENSION:

f_t= Bolt Tensile Stress = 4.079ksi (from page C-12)
T=Bolt Tension=4.079x1.767=7.21 k/bolt
L=Embedment=20", C=Bolt Circumference=4.71"
U=Allowable Bond Stress Concrete To Plain Bar
U=0.09ksi (3000psi Concrete)
u=Actual Bond Stress=T/CL
u=7.21/(20x4.71)=0.0765ksi < U=0.09ksi

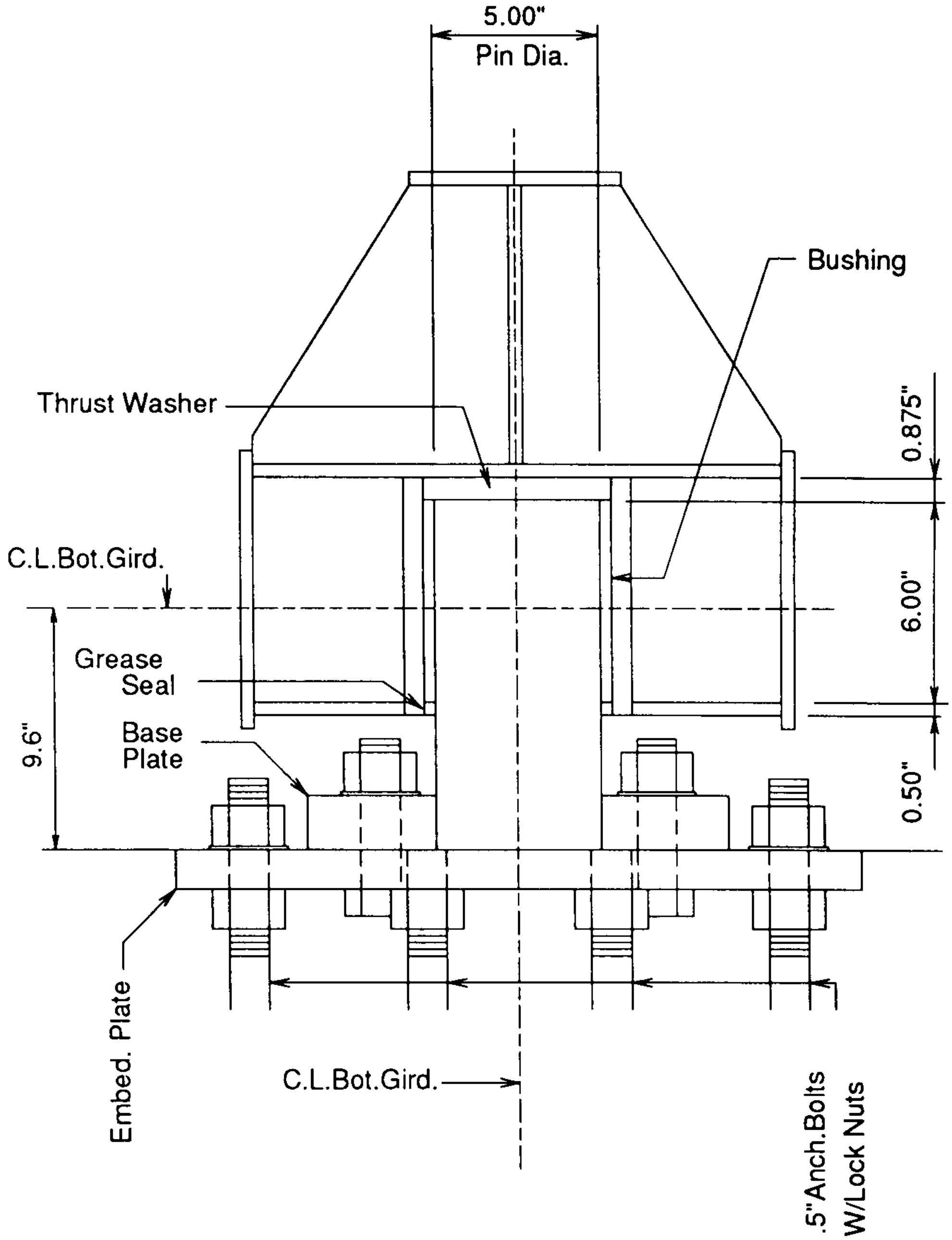
BOND STRESS ALONE IS SUFFICIENT TO RESIST ANCHOR BOLT TENSION; HOWEVER, BOLTS SHALL HAVE 90 DEGREE HOOKS WITH 6" LEGS.

BUSHING DESIGN : (Mat'l. ASTM B22)

F_p=Max.Allowable Avg. Brg. Stress=1.11x3.00=3.33ksi

L_{min}=P/(F_pxD)=55.79/(3.33x5)=3.35" USE L=6"

See Page B-15 For Calculation Of Actual Max. Brg. Stress.



SECTION THRU BOTTOM HINGE

CALCULATION FOR ACTUAL MAXIMUM BEARING PRESSURE-BOTTOM PIN

L = Bushing Length = 6", Nominal Pin Diameter = 5"

R1 = Min. Radius Of Pin = 2.4985"

R2 = Max. Inside Radius Of Bushing = 2.5010"

E1 = Modulus Of Elasticity Of Pin = 29000ksi

E2 = Modulus Of Elasticity Of Bushing = 15000ksi

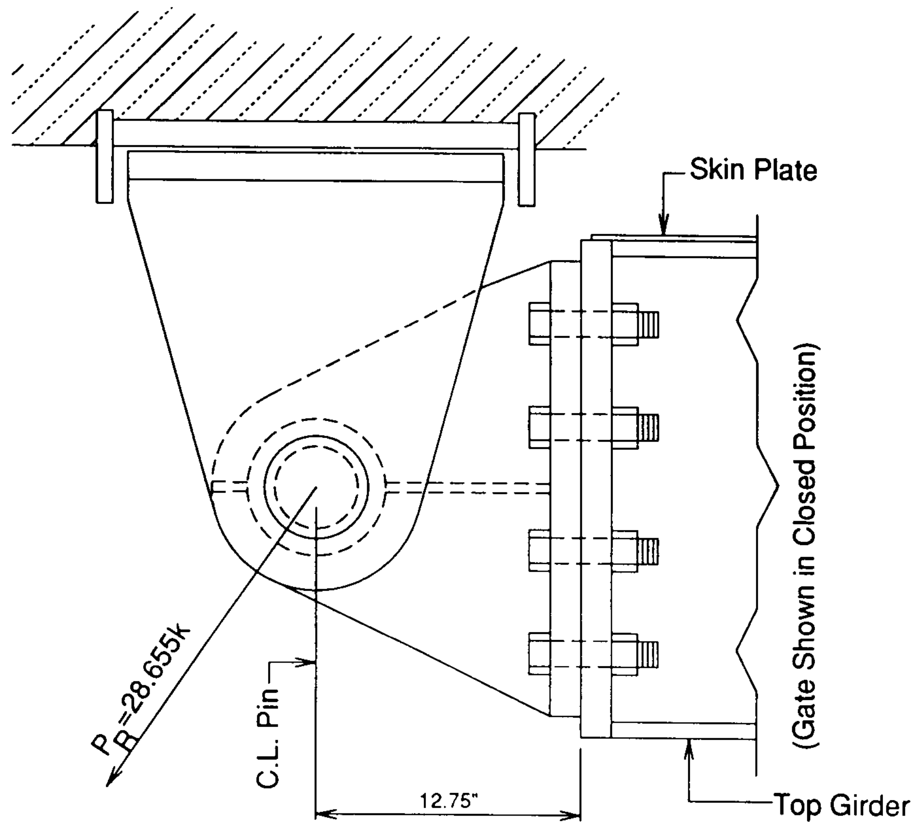
P1 = 55.79/6 = 9.30k/in

$$f_{p\ max} = 0.591 \sqrt{\frac{P1\ E1\ E2\ (R2-R1)}{(E1+E2)(R1R2)}}$$

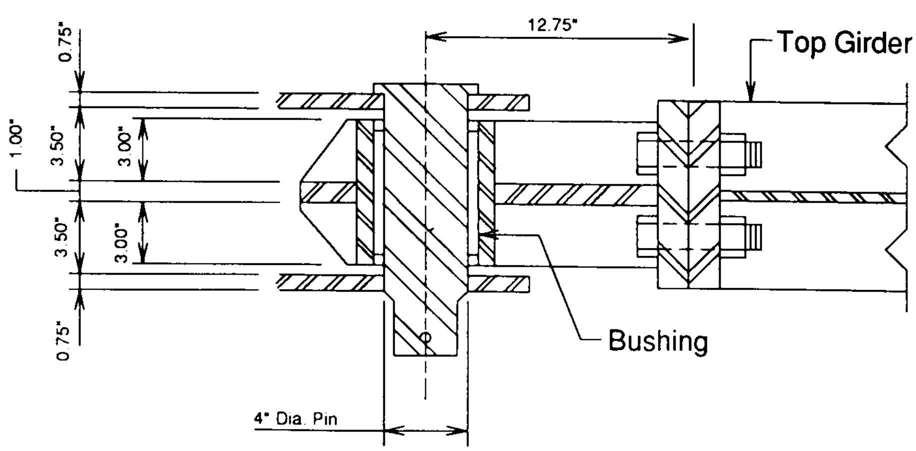
$$f_{p\ max} = 0.591 \sqrt{\frac{9.3 \times 435 \times 10^{6} \times 0.0025}{44 \times 10^{3} \times 6.24875}} = 3.584\text{ksi}$$

A Maximum Bearing Stress Equal To Or Less Than The Yield Strength Of The Material Is Allowable.

TOP HINGE:



PLAN



SECTION THRU HINGE

TOP PIN (Mat'l. ASTM A276, Fy = 55ksi)

P_R= 28.655k, V = 28.655/2 = 14.33k (Dbl. Shear)

M = 28.655x8.75/4 = 62.68k-in

A = $3.1416(4)^2/4$ = 12.57, S = $3.1416(4)^3/32$ = 6.28

F_b =1.11(0.75F_y)=45.83ksi

F_V = 1.11(0.4F_y) = 24.44ksi

f_V=14.33/12.57 = 1.14ksi < 24.44ksi o.k.

f_b= 62.68/6.28 =9.98ksi < 45.83ksi o.k.

TOP BUSHING (Mat'l. ASTM B22)

P_R = 28.655k, Bushing Length = 6.00"

Inside Dia. = 4.00"

F_p = Avg. Allowable Brg. Stress =1.11x3.00 = 3.33ksi

f_p= 28.655/(4x6) = 1.194ksi < 3.33ksi o.k.

CHECK MAX. BEARING STRESS:

R1 = 1.997", R2 = 2.001"

E1 = 29000ksi, E2 = 15000ksi

P1 = 28.655/6 = 4.78k/in

$$\text{(Max)} \quad f_p = 0.591 \sqrt{\frac{4.78 \times 29 \times 15 \times 10^6 (2.001-1.997)}{(29+15)(10^3)(2.001 \times 1.997)}} = 4.065\text{ksi}$$

A Maximum Bearing Stress Equal To Or Less Than The Yield Strength Of The Material Is Allowable.

Check Bottom Girder Design With Axial Load & Bending Due To Diagonal Included.

P = Axial Load From Diagonal = 7.808 k (See page C-8)
e = Eccentricity = 10.674", Pe = 83.34 k-in
M(Hs) = Moment Due To Hydrostatic Load = 4992 k-in (See page C-7)
M = Pe + M(Hs) = 83.34 + 4992 = 5075.34 k-in

Properties of W24×84:

$$A = 24.7,\ S = 196,\ r_x = 9.79,\ \frac{b_f}{2t_f} = 5.9,\ \frac{d}{t_w} = 51.3$$

Member is Compact (Table B5.1) $L_x = 379"$, $L_b = 66"$

$$\frac{KL_x}{r_x} = 39,\ f_a = \frac{7.808}{24.7} = 0.316 \text{ ksi},\ F_a(AISC) = 19.27 \text{ ksi}$$

$$F_a = 1.11F_a(AISC) = 21.41 \text{ ksi},\ F_b = \frac{M}{S} = 25.89 \text{ ksi}$$

$$\frac{76b_f}{\sqrt{F_y}} = 114'' > L_b = 66''\ ,\ F_b(AISC) = 24 \text{ ksi},\ F_b = 1.11\times 24 = 26.67 \text{ ksi}$$

$$\frac{f_a}{F_a} < 0.15,\ \frac{f_a}{F_a} + \frac{f_b}{F_b} = < 1.00\ (H1\text{-}3)$$

$$\frac{0.316}{21.41} + \frac{25.89}{26.67} = 0.986 < 1.00$$

W24×84 Satisfies Axial Load & Flexural Requirements.